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Research Article

Examination of strength reduction factor in CFRP-confined columns under axial compression

Aysun TEKİN ÖZER[1], Sema ALACALI[1,*]

[1] *Department of Civil Engineering, Yildiz Technical University, Istanbul, 34220, Türkiye*

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ABSTRACT

In the strength design method, the safety level is tried to be reached by using strength reduction factors ϕ applied to the nominal strengths. In the current ACI 318-19, the safety factors ϕ are suggested as 0.65 for tie-reinforced concrete columns and 0.60 for plain columns, respectively. The aim of the present study is to evaluate the strength reduction factors of the confined columns with Carbon Fiber Reinforced Polymer (CFRP) subjected to axial compression load according to ACI 318-19. For this purpose, a total of 298 column test specimens with circular and rectangular cross-sections confined with CFRP collected from 18 different experimental studies in the literature were examined to determine the strength reduction factors. The first-order second-moment approach was used to determine these factors by reliability analysis, and the correlation effects between random variables were not taken into account. The target reliability index and the corresponding probability of failure were taken as β=3.5 and p_F = 2.33×10^{-4} for different coefficients of variation of random variables, respectively. Then, these factors were been separately compared to those of recommended in the ACI 318-19. At the end of the study, the upper limits of the coefficients of variation that should not be exceeded were determined to ensure the target reliability of the columns.

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INTRODUCTION

Today, one of the most frequently used method in the field of structural strengthening is to wrap the column with fiber polymers. Fiber polymers act like transverse reinforcement in reinforced concrete columns, causing an increase in the axial load-carrying capacity and ductility of the columns. High strength capacity, high resistance to corrosion, light weight, ease of application, not requiring costly equipment, being applicable to any shape due to its flexibility, and providing the opportunity to be applied on buildings without distrupting the existing use are among the most important advantages of fiber polymers [1].

One of the main features of engineering design is to provide structural safety by considering economic conditions. The reliability of a structural element can be explained as the probability of continuing its intended use throughout its lifespan. Safety factors were recommended from reliability analysis for the design purposes which ensure structural

***Corresponding author.**
*E-mail address: noyan@yildiz.edu.tr

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safety. Depending on the β, the safety factors are determined and the desired safety level is aimed to be achieved. Reliability and probability of failure are directly related to the risk taken into account in determining the safety factor [2].

In ACI 318-19, the reliability of structural elements are obtained by multiplying the strength values with a strength reduction factor ϕ which is less than one. The ϕ is used to take into account uncertainties in both the size and strength of the material to reduce possible design equation errors [1]. In the structural reliability, ϕ depends on the accepted target reliability index β. β also varies depending on the accepted failure probability p_F in the structural design. As per ACI 318-19 code recommendation, β taken into account in determining of the ϕ is taken as 3.5, and the failure probability p_F corresponding to this value is taken as 2.33×10^{-4} in the current study.

The ϕ values in the ACI 318-19 for the axial compression load are different for plain concrete column and both types of transverse reinforcing steels (spiral or tie) in column. In ACI 318-19, the ϕ factor is suggested as 0.75 for spiral reinforced concrete columns, 0.65 for tie reinforced concrete columns and 0.60 for plain concrete columns, respectively [1]. ACI 440.2R-17 emphasizes that strength reduction factor for CFRP-confined RC columns under the axial compression load should be used as required by ACI 318-19 [3].

In the literature, a study was investigated to determine the strength reduction factor of axially-loaded CFRP-confined columns by Ozer and Alacali [4]. The factors ϕ were determined for CFRP-confined specimens with 21 transverse and longitudinal reinforcement and 38 plain circular columns under axial load collected from literature in this study. Then, these factors were compared those of recommended in the ACI 318-19.

In the current study, however, the strength reduction factors for different coefficients of variation were determined by considering the axial compression test data of 298 circular and rectangular column specimens with/out longitudinal reinforcement collected from 18 different papers in the literature by increasing the number of experiments. Then, the obtained factors are been compared to those of recommended in the ACI 318-19. To determine the strength reduction factors, the performance functions were obtained from the equations in ACI 440.2R-17.

The first-order second-moment (FOSM) approach, which is one of the probabilistic methods, was utilized to determine these factors and it was assumed that the random variables constituting the performance functions were statistically independent. In other words, the correlation effects between the variables were neglected. The effects on the reduction factors of the columns of the variability in the coefficients of variation of the random variables in performance function of CFRP-confined columns under axial compression load were examined. In order to achieve an acceptable safety level, the coefficients of variation corresponding to the factors recommended in ACI 318-19 have been determined.

DESIGN OF CFRP-CONFINED REINFORCED CONCRETE COLUMN

According to ACI 318-19, the basic requirement in the structural design is expressed as follows:

$$\phi P_n \geq P_u \tag{1}$$

The design strength (ϕP_n) obtained by multiplying with a factor (ϕ) must be sufficient to satisfy the required strength P_u. The axial load carrying capacities of the fiber polymer-wrapped columns for both types of transverse reinforcing steel (spirals or ties) can be calculated with the following equations according to the ACI 440.2R-17. As seen in Equations (2) and (3), any contribution of lateral reinforcement to the axial compression strength of a reinforced concrete column is not taken into account [3].

For steel spiral reinforcement;

$$\phi P_n = 0.85\phi\ [0.85 f'_{cc}(A_g - A_{st}) + f_y A_{st}] \tag{2}$$

For steel-tie reinforcement;

$$\phi P_n = 0.80\phi\ [0.85 f'_{cc}(A_g - A_{st}) + f_y A_{st}] \tag{3}$$

where, in above equations, f'_{cc} is compressive strength of FRP-confined concrete, A_g is gross area of concrete section, A_{st} is total area of longitudinal reinforcement, f_y is yield strength of longitudinal reinforcement, ϕ is strength reduction factor and P_n is nominal axial compressive strength [3].

The compressive strength of FRP-confined concrete f'_{cc} and the maximum confinement pressure of FRP jacket f_l can be written as:

$$f'_{cc} = f'_c + \psi_f 3.3 \kappa_a f_l \tag{4}$$

$$f_l = \frac{2 n t_f E_f \varepsilon_{fe}}{D} \tag{5}$$

where, f'_c is the compressive strength of unconfined concrete, κ_a is the shape factor, n is the number of plies of FRP reinforcement, t_f is the nominal thickness of one ply of FRP reinforcement, E_f is the modulus of elasticity of FRP, ε_{fe} is the effective strain in FRP reinforcement attained at failure, D is the diameter of circular section or diagonal distance equal to $D = \sqrt{b^2 + d^2}$ for rectangular section as seen in Figure 1. The shape factor κ_a in Equation 4 for circular section is taken as 1 [3]. For rectangular section, κ_a is calculated depending on the effective confinement area A_e. The values of the b, h and A_c seen in Equation 6 are cross-sectional dimensions and area of the column, respectively.

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 \tag{6}$$

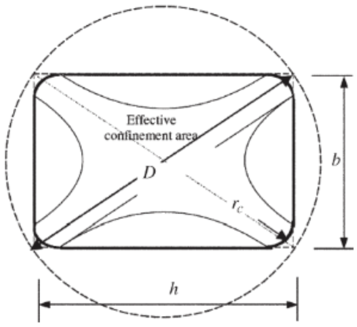


Figure 1. Equivalent circular cross section [3].

In Equation 4, "Ψ_f" is defined as additional reduction factor and taken as 0.95 in ACI 440.2R-17. The ε_{fe} in Equation 5 means the strain efficiency factor which is used to consider the premature failure of FRP in relation to the stress concentration regions caused by cracking of concrete as it expands. ε_{fe} is as given below in ACI 440.2R-17:

$$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu} \tag{7}$$

Various experimental calibrations were realized to determine the strain efficiency factor κ_ε, and the mean value of κ_ε was proposed as 0.586 by Lam and Teng and 0.58 by Harries and Carey [3]. In this study, it was assumed that κ_ε equals to 0.586. In Equation 7, ε_{fu} is defined as design rupture strain of FRP reinforcement and calculated by Equation 8. C_E in Equation 8 is environmental reduction factor and ACI 440.2R-17 recommends 0.95 for structures exposed to interior environmental conditions. In Equation 8, ε^*_{fu} is defined as ultimate rupture strain of CFRP.

$$\varepsilon_{fu} = C_E \varepsilon^*_{fu} \tag{8}$$

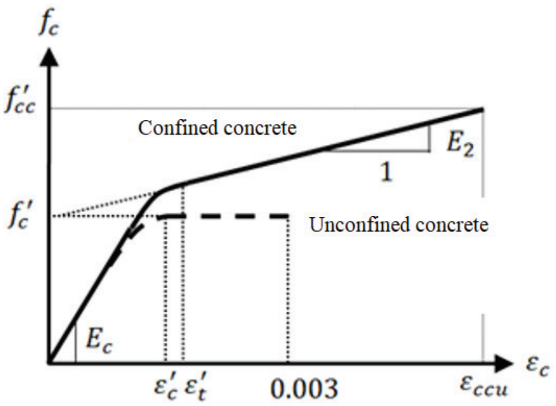


Figure 2. Stress-strain model for FRP-confined concrete [3].

As shown in Figure 2, according to this proposed model in the ACI 440.2R-17, E_c represents the modulus of elasticity of concrete, ε_c', ε_t' and E_2 represent compressive strain corresponding to f_c', transition strain and slope of the linear component of the stress-strain model, respectively. The ultimate axial strain of confined concrete ε_{ccu} is calculated by Equation 9. In addition, ε_{ccu} calculated by the Equation 9 should be equal or less than 0.01 to avoid concrete integrity being lost due to excessive cracking as shown in Equation 10 [3].

$$\varepsilon_{ccu} = \varepsilon_c' \left[1.50 + 12\kappa_b \frac{f_l}{f_c'}\left(\frac{\varepsilon_{fe}}{\varepsilon_c'}\right)^{0.45}\right] \tag{9}$$

$$\varepsilon_{ccu} \leq 0.01 \tag{10}$$

In case ε_{ccu} calculated by the Equation 9 is greater than 0.01, a new limit value ($\varepsilon_{ccu.new} = 0.01$) is taken. In this case, ($f'_{cc.new}$) is recalculated with the following equations.

$$f'_{cc,new} = f_c' + E_2 \varepsilon_{ccu,new} \tag{11}$$

$$f'_{cc,new} = f_c' + 0.01 \frac{f'_{cc} - f'_c}{\varepsilon_{ccu}} \tag{12}$$

The accuracy of the above-mentioned all equations for specimens with unconfined concrete compressive strength of 70 MPa and greater was not been experimentally proven. Also, ACI 440.2R-17 does not recommend for rectangular cross section with side aspect ratios h/b greater than 2.0, or face dimensions b or h exceeding 900 mm. Therefore, the aforementioned specimens are not considered in current study.

RELIABILITY ANALYSIS

Partial Safety Factors

As statistical information about the random variables that constitute the performance function is usually limited to the mean values (first-moment) and variances (second-moment). For this reason, reliability equations include terms based on the first and second moment of random variables. In the probabilistic design with the second moment approach, different safety factors can be determined for each design variable. The designs corresponding to different failure surfaces of reduced variables can be represented as follows (Figure 3) [2]:

$$g\left(\gamma_1 m_{x_1}, \gamma_2 m_{x_2}, \ldots \gamma_i m_{x_i}, \ldots . \gamma_n m_{x_n}\right) = 0 \tag{13}$$

Using the most probable failure point ($x_i^* = \gamma_i m_{x_i}$) on the failure surface, the partial safety factors γ_i are determined by the following Equation [2].

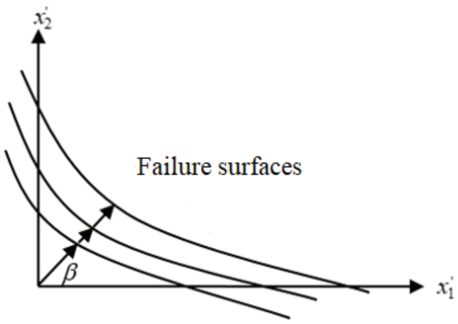


Figure 3. Designs corresponding to different failure surfaces [2].

$$\gamma_i = \frac{x_i^*}{m_{x_i}} \tag{14}$$

Similarly, the sensitivity coefficient (α_i^*) obtained from the most likely failure point (x_i^* = -$\alpha_i^* \beta$) in the space of reduced variables is given by Equation (15) [2].

$$\alpha_i^* = \frac{\left(\dfrac{\partial g}{\partial X_i}\right)_*}{\left[\sum_{i=1}^{n} \left(\dfrac{\partial g}{\partial X_i}\right)_*^2\right]^{\frac{1}{2}}} \tag{15}$$

In Equation 14, the original variables x_i^* are expressed by the following equation:

$$x_i^* = m_{x_i} - \alpha_i^* \beta \sigma_{x_i} = m_{x_i}(1 - \alpha_i^* \beta V_{x_i}) \tag{16}$$

By writing Equation 16 instead of x_i^* in Equation 14, the γ_i safety factors are;

$$\gamma_i = \frac{x_i^*}{m_{x_i}} = \frac{m_{x_i}(1 - \alpha_i^* \beta V_{x_i})}{m_{x_i}} = 1 - \alpha_i^* \beta V_{x_i} \tag{17}$$

If the distributions of variables that constitute the performance function are not normal or the structure of the performance function is non-linear, iteration is required to reveal x_i^*. In the case of the lognormal or Type-I asymptotic distributions, these distributions are transformed to equivalent normal distributions [2,5].

Performance Functions

The performance function for axially loaded columns is calculated with the following equation;

$$g(X) = R - S = \gamma_1 P_n - \gamma_2 P_u = \phi P_n - P_u \tag{18}$$

In this study, P_u and P_n represent the axial compressive strengths obtained from the experimental data and equation proposed in ACI 440.2R, respectively. γ_1 and γ_2 are the safety factors of the variables corresponding to β. It is supposed that the safety factor γ_1 corresponds to ϕ calculated according to probabilistic theory. Then, the factors obtained from probabilistic calculations are compared with the values recommended in ACI 318-19.

In ACI 318-19, ϕ is proposed as 0.75 for axially loaded spiral reinforced concrete columns, 0.65 for other axially-loaded reinforced concrete columns and 0.60 for axially-loaded unreinforced columns. As mentioned in previous sections, the value of the ϕ also varies depending on the β. In the literature, β is proposed as 3.5 by Zou and Hong [6], Alqam et al. [7], and Zhou et al. [8]. Mirza [9] suggested the β between 3.0-3.5 in his study on transversely reinforced concrete columns. The β can be considered as a function of the failure probability p_F as follows:

$$\beta = -\Phi^{-1}(p_F) \tag{19}$$

where Φ^{-1} is the inverse of the cumulative distribution function. In this study, β taken into account in determining of the ϕ is considered as 3.5, and the failure probability corresponding to this value is calculated as $p_F = 2.33 \times 10^{-4}$. Making necessary arrangements in Equation (3), the performance functions were obtained by Equations (20)-(23).

For FRP-confined reinforced concrete circular column is as given below:

$$g(X) = \phi \left[0.80\left(f_c' + \frac{3.49 n t_f E_f \varepsilon_{fu}^*}{D}\right)\left(\frac{\pi D^2}{4} - \frac{\rho_g \pi D^2}{4}\right) + f_y \frac{\rho_g \pi D^2}{4}\right] - P_u \tag{20}$$

For FRP-confined unreinforced circular column is as given below:

$$g(X) = \phi \left[0.80\left(f_c' + \frac{3.49 n t_f E_f \varepsilon_{fu}^*}{D}\right)\frac{\pi D^2}{4}\right] - P_u \tag{21}$$

For FRP-confined reinforced rectangular column is as given below:

$$\begin{aligned} g(X) = \phi \Bigg[& 0.80\left(f_c' + \frac{3.49 n t_f E_f \varepsilon_{fu}^*}{\sqrt{b^2 + h^2}}\left(\frac{1 - \frac{\left[\left(\frac{b}{h}\right)(h - 2r_c)^2 + \left(\frac{h}{b}\right)(b - 2r_c)^2\right]}{3(bh - 4r_c^2 + \pi r_c^2)} - \rho_g}{(1 - \rho_g)}\right)\frac{b^2}{h^2}\right)\left((bh - 4r_c^2 + \pi r_c^2)\right. \\ & \left. - \rho_g(bh - 4r_c^2 + \pi r_c^2)\right) + f_y \rho_g (bh - 4r_c^2 + \pi r_c^2)\Bigg] - P_u \end{aligned} \tag{22}$$

For FRP-confined unreinforced rectangular column is as given below:

$$g(X) = \phi \left[0.80\left(f_c' + \frac{3.49 n t_f E_f \varepsilon_{fu}^*}{\sqrt{b^2 + h^2}}\left(1 - \frac{\frac{b}{h}(h - 2r_c)^2 + \frac{h}{b}(b - 2r_c)^2}{3(bh - 4r_c^2 + \pi r_c^2)}\right)\frac{b^2}{h^2}\right)(bh - 4r_c^2 + \pi r_c^2)\right] - P_u \tag{23}$$

Coefficients of Variation and Distribution Types of Variables

All experimental parameters regarding the CFRP-confined column specimens were modeled as random variables in order to provide a probabilistic analysis. The statistical parameters of variables to determine the strength reduction factor were investigated in accordance with the studies available by reviewing the literature and codes.

The $V_{f_c'}$ was taken between 0.1 and 0.2 through the literature depending on the concrete grade. The $V_{f_c'}$ was suggested 0.11 by Hao et al.[10], 0.16 by Ali [11], 0.12 by Jafari [12], 0.13 by Val et al. [13], 0.15 by Atadero and Karbhari [14], 0.18 by Hong and Zhou [15], 0.2 by Monti and Santini [16], 0.18 by Taki et al. [17], 0.18 by Wieghaus and Atadero [18], 0.18 by Okeil et al. [19], and 0.15 by Ghobarah et al. [20]. In this study, it is used 0.10, 0.12 and 0.15 for $V_{f_c'}$.

The V_{f_y} varies between 0.04 and 0.15. The V_{f_y} was proposed as 0.1 by Ali [11], 0.1 by Jafari [12], 0.1 by Val et al. [13], 0.098 by Hong and Zhou [15], 0.15 by Monti and Santini [16], 0.04 by Kim et al. [21], 0.11 by Ellingwood [22], 0.125 by Taki et al. [17], 0.093 by Wieghaus ve Atadero [18], 0.125 by Okeil et al. [19], 0.05 by Huang et al. [23], 0.098 by Stewart and Attard [24], 0.93-0.107 by Ghobarah et al. [20], and 0.05 by Mestrovic et al. [25]. In this study, it is used 0.10 for V_{f_y}.

The V_{E_f} varies between 0.04 and 0.15. The V_{E_f} is proposed as 0.2 by Atadero and Karbhari [14], 0.15 by Taki et al. [17] and 0.1 by Wieghaus and Atadero [18]. In this study, it is used 0.10, 0.15 and 0.20 for V_{E_f}. The $V_{\varepsilon_{fu}^*}$ was proposed as 0.022 in literature. The value of $V_{\varepsilon_{fu}^*}$ was taken 0.2 by Taki et al. [17] and Okeil et al. [19]. In this study, it is used 0.022 for $V_{\varepsilon_{fu}^*}$. The V_{t_f} varies between 0.05 and 0.07. The V_{t_f} is proposed as 0.07 by Hao et al. [10], 0.05 by Ali [11], 0.05 by Atadero and Karbhari [14], 010 by Taki et al. [17] and 0.05 by Wieghaus and Atadero [18]. In this study, it is used 0.05 and 0.07 for V_{t_f}.

The coefficients of variation of V_D, V_b, V_h and V_{r_c} vary between 0.01 and 0.05. The V_D is proposed as 0.03 by Hao et al. [10], Wieghaus and Atadero [18] and Okeil et al. [19] and 0.02 by Ghobarah et al. [20]. In this study, it is used 0.03 for the coefficient of variation of V_D, V_b, V_h and V_{r_c}. The V_{ρ_g} was proposed as 0.01 by Hao et al. [10]. Therefore, it is used 0.01 for V_{ρ_g} in this study.

Besides, the probability distribution types of the variables that constitutive the performance function are one of the important factor in the probability theory to determine

Table 1. Coefficients of variation and distribution types of variables

Cases	$V_{f_c'}$	V_{E_f}	V_{t_f}	V_{f_y}	$V_{\varepsilon_{fu}^*}$	V_D, V_b, V_h, V_{r_c}	V_{ρ_g}	V_{pu}
1	0.10	0.10	0.05	0.10	0.022	0.03	0.10	0.01
2			0.07					
3		0.15	0.05					
4			0.07					
5		0.20	0.05					
6			0.07					
7	0.12	0.10	0.05					
8			0.07					
9		0.15	0.05					
10			0.07					
11		0.20	0.05					
12			0.07					
13	0.15	0.10	0.05					
14			0.07					
15		0.15	0.05					
16			0.07					
17		0.20	0.05					
18			0.07					
19	0.18	0.10	0.05					
20			0.07					
21		0.15	0.05					
22			0.07					
23		0.20	0.05					
24			0.07					
D.T*	LD*	LD*	ND*	LD*	T-1 AD*	ND*	ND*	T-1 AD*

DT*:Distribution Type, LD*:Lognormal Distribution, ND*:Normal Distribution, T-I AD*: Type I Asymptotic Distribution the strength reduction factors. In this regard, according to international statistics, normal distribution for variables such as cross section, proportional expressions, area and perimeter; normal or preferably lognormal distribution for material strength, modulus of elasticity; normal distribution for time-invariant loads; Type-I asymptotic distribution for time varying loads; Type-I or Type-II asymptotic distribution for wind and earthquake loads; For snow loads, Type-I asymptotic or Weibull distributions are used [2].

The distributions of the f_c', f_y and E_f variables are considered as log-normal in this study. Normal distribution is used for t_f, ρ_g, D, b, h and r_c. Type-I asymtotic distribution is used for the ε_{fu}^* and P_u. Table 1 represents the coefficient of variation and distribution type for each variable considered in the present study.

EXPERIMENTAL DATABASE

In this study, an extensive database of 298 column test specimens of circular and rectangular cross-section, with/without longitudinal and transverse reinforcement collected from 18 different experimental studies in the literature is taken into account to determine the strength reduction factors. All database includes specimens subjected to monotonic load. Besides, specimens with partial CFRP confinement were not taken into account. The properties of the column test specimens in collected database are given in the following sections.

Test Specimens with Circular Cross-section

A total of 21 column specimens, collected from 4 different studies, were examined in the group of circular cross-sectional columns CFRP-confined with longitudinal reinforcement. One of specimens in the database is high-strength concrete (HSC), and the remaining 20 specimens are normal-strength concrete (NSC). CFRP jacket was applied in all test specimens. A total of 164 column specimens obtained from 8 different studies are examined in the group of circular columns CFRP-confined without longitudinal reinforcement. 36 specimens are HSC, and the remaining 128 specimens are NSC. CFRP tube was used in 23 of the specimens and the in the remaining 141 specimens was used CFRP jacket. The mechanical and geometrical properties of the circular test specimens are given in Table 2. Moreover, Figure 4 shows frequency distributions of the input variables for circular test specimens.

Test Specimens with Rectangular Cross-section

A total of 41 column specimens obtained from 5 different studies were examined in the rectangular cross-sectional column group with longitudinal reinforcement. 3 specimens are HSC, the remaining 38 specimens are NSC. CFRP jacket was applied in all specimens. A total of 72 specimens CFRP-confined obtained from 5 different studies were examined in the rectangular cross-sectional column group without longitudinal reinforcement. 15 of them are HSC and the remaining 57 specimens are NSC. 3 of the test specimens were confined by CFRP tube, the remaining 69 specimens were confined with CFRP jacket. The mechanical and geometrical properties of the rectangular test specimens are summarized in Table 3. Figure 5 shows frequency distributions of the input variables for rectangular test specimens.

EXAMINATION OF STRENGTH REDUCTION FACTORS

FRP-confined Circular Columns

As a result of the probabilistic calculations, it is shown that the (ϕ=0.65) for transversely reinforced columns in ACI 318-19 corresponds to the case "23" ($V_{f_c'}$ = 0.18, V_{t_f}= 0.05, V_{E_f} = 0.20, $V_{\varepsilon_{fu}^*}$ = 0.022, V_D = 0.03, V_{ρ_g} = 0.010, V_{f_y} = 0.10) for the (β=3.5). The strength reduction factors ϕ for each case shown in Table 4a were the means of the values obtained from 21 test data. The reduction factors ranged from 0.607~0.721, and in approximately 62% of the specimens, the reduction factors are lower than the (ϕ=0.65)

Table 2. Mechanical and geometrical properties of the circular test specimens

Reference	No.	Fiber Type	Concrete Type	Confinement Type	D (mm)	H (mm)	f'_c (MPa)	f_y (MPa)	ρ_g	Steel Type	n	t_f (mm)	E_f (Gpa)	ε_{fu}^* (mm/mm)	P_u (kN)
Chastre and Silva [26]	8	CFRP	NSC	Jacket	150-250	750	35.2-38.0	391-458	0.0096-0.0138	Tie	1-4	0.167-0.176	226-241	0.014-0.015	1375.80-4828.30
Benzaid and Mesbah [27]	4	CFRP	NSC-HSC	Jacket	160	320	29.51-61.81	500	0.0225	Tie	1-3	0.13	238	0.018	1206.53-2199.63
Faustino et al. [28]	3	CFRP	NSC	Jacket	150	750	34.60	587	0.0128	Tie	2	0.176	217	0.016	1804.0-1843.0
Peker [29]	6	CFRP	NSC	Jacket	250	500	12.84-13.53	367	0.0096	Tie	1-5	0.165	230	0.015	1584.76-4675.29
Lin and Li [30]	27	CFRP	NSC	Jacket	100-150	200-300	17.05-24.93	-	-	-	1-3	0.11	232	0.018	348.15-1508.44
Karabinis and Rousakis [31]	18	CFRP	NSC	Jacket	200	320	35.7-38.5	-	-	-	1-3	0.117	240	0.016	1306.90-2120.58
Vincent and Ozbakkaoglu [32]	14	CFRP	NSC-HSC	Jacket	152	305	35.5-65.8	-	-	-	1-4	0.117	240	0.016	782.09-1892.61
Benzaid and Mesbah [27]	4	CFRP	NSC-HSC	Jacket	160	320	25.93-61.81	-	-	-	1-3	1.00	34	0.014	796.81-1873.70
Ozbakkaoglu and Vincent [33]	23	CFRP	NSC-HSC	Tube	74-302	150-605	34.6-66.6	-	-	-	1-4	0.117	240	0.0155	261.92-4082.99
Berthet et al. [34]	24	CFRP	NSC-HSC	Jacket	160	320	22.2-52.1	-	-	-	1-12	0.11-0.165	240	0.0155	760.01-3343.66
Wu and Jiang [35]	34	CFRP	NSC	Jacket	150	300	20.6-36.7	-	-	-	1-4	0.167	242	0.0171	889.76-2505.28
Theodoros and Tepfers [36]	20	CFRP	NSC-HSC	Jacket	150	300	25.2-51.8	-	-	-	1-3	0.17	377	0.012	685.65-2238.97

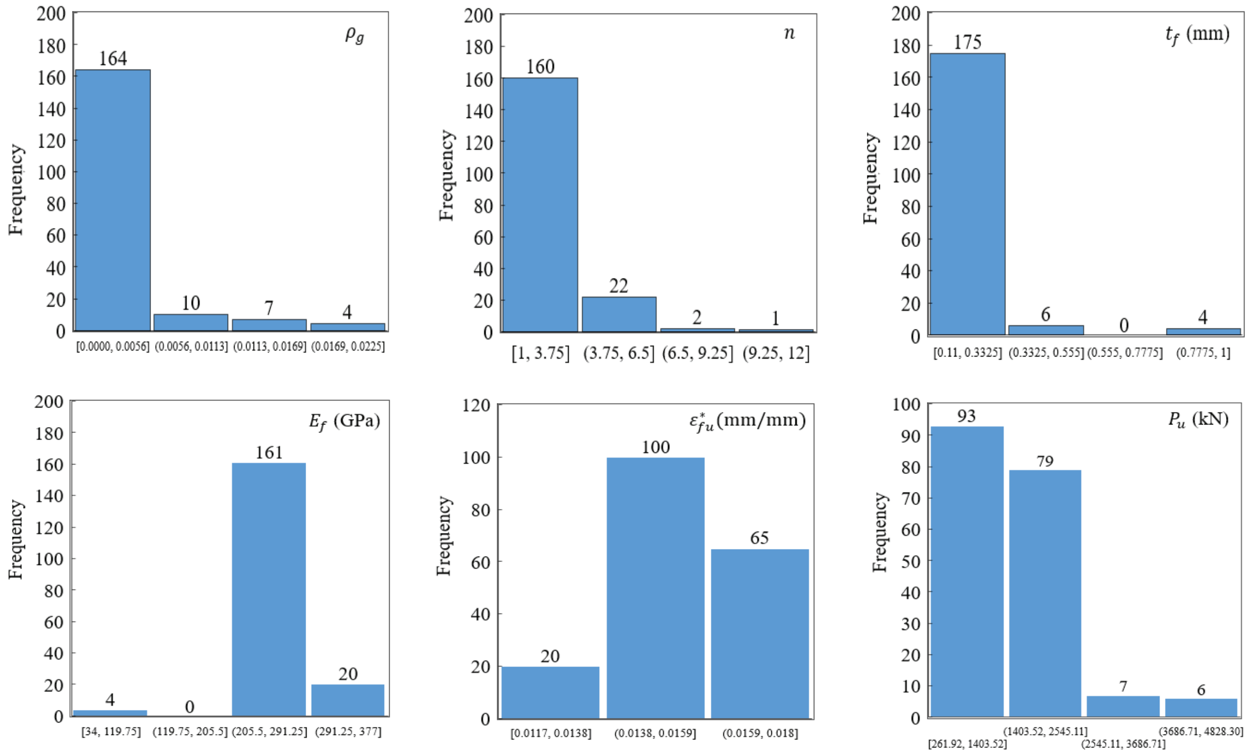


Figure 4. Frequency distributions of the input variables for circular test specimens.

Table 3. Mechanical and geometrical properties of the rectangular test specimens

Reference	No.	Fiber Type	Concrete Type	Confinement Type	b (mm)	h (mm)	H (mm)	r_c (mm)	f'_c (MPa)	f_y (MPa)	ρ_g	Steel Type	n	t_f (mm)	E_f (GPa)	ε^*_{fu} (mm/mm)	P_u (kN)
Benzaid and Mesbah [27]	4	CFRP	NSC-HSC	Jacket	140	140	280	0	24.77-59.53	-	-	-	1-3	1.00	34	0.014	542.14-1378.86
Faustino et al. [28]	9	CFRP	NSC	Jacket	150	150	750	0-38	34.6	587	0.01	Tie	2	0.176	217	0.0155	751.00-1416.00
Peker [29]	8	CFRP	NSC	Jacket	150-250	250-300	500	40	13.53	339-345	0.01	Tie	1-5	0.165	230	0.015	946.90-2473.12
Zeng et al. [37]	7	CFRP	NSC	Jacket	290	435	1300	25-65	30.8-37.4	491.4	0.0149	Tie	2-6	0.334	245.6	0.0171	6661.38-9709.48
Wang et al. [38]	8	CFRP	NSC	Jacket	204-305	204-305	612-915	20-30	25.50	312-340	0.015	Tie	1-3	0.167	240	0.018	1594.20-3715.50
Belouar et al. [39]	9	CFRP	NSC-HSC	Jacket	140	140	280-1000	0	24.69-69.98	500	0.0225	Tie	1-3	0.13	238	0.018	875.68-1804.72
Al-Salloum [40]	8	CFRP	NSC	Jacket	150	150	500	5-50	26.72-31.82	-	-	-	1.0	1.2	75.1	0.0125	925.67-1296.14
Ozbakkaloglu and Oehlers [41]	4	CFRP	NSC	Tube-Jacket	150-200	200-300	600	20-40	25.8-28.0	-	-	-	3-5	0.117	240	0.0155	1305.58-1823.17
Rochette and Labossiere [42]	12	CFRP	NSC	Jacket	152-203	152	500	5-38	35.8-43.9	-	-	-	3-5	0.3	82.7	0.015	976.27-1502.88
Wang and Wu [43]	44	CFRP	NSC-HSC	Jacket	150	150	300	0-60	29.3-55.2	-	-	-	1-2	0.165	230-230.5	0.015	687.05-1764.34

value proposed in ACI 318-19 while approximately 38% of the specimens meet the regulations.

The value of (ϕ=0.60) proposed for unreinforced/plane columns in ACI 318-19 corresponds to the case "24" ($V_{f'_c}$ = 0.18, V_{t_f} = 0.05, V_{E_f} = 0.20, $V_{\varepsilon^*_{fu}}$ = 0.022, V_D = 0.03, V_{ρ_g} = 0.010, V_{f_y} = 0.10) for the (β=3.5) as seen in Table 4b. The mean values of the ϕ obtained from 164 test data for each case are shown in Table 4b. The reduction factors ϕ ranged from 0.596~0.693, and in approximately 46% of the specimens, the values of ϕ are lower than the proposed value (ϕ=0.60) in ACI 318-19 while approximately 54% of the specimens meet the regulations [44].

As seen in Table 4, assuming that the coefficients of variation of other variables remain constant, the ϕ decreases with the increase of the variation coefficients of the f'_c, E_f and t_f for the reinforced and unreinforced column specimens. It is seen that the effect of the uncertainty in the concrete strength on ϕ for the examined column specimens is higher than those of the other variables.

In Figure 6, the variations of the strength reduction factors for 24 different cases have also shown for the reinforced and unreinforced circular test specimens, respectively. According to this distributions, as the coefficients of variation of the variables increase, the strength reduction factors decrease. Namely, the increase in the uncertainty in

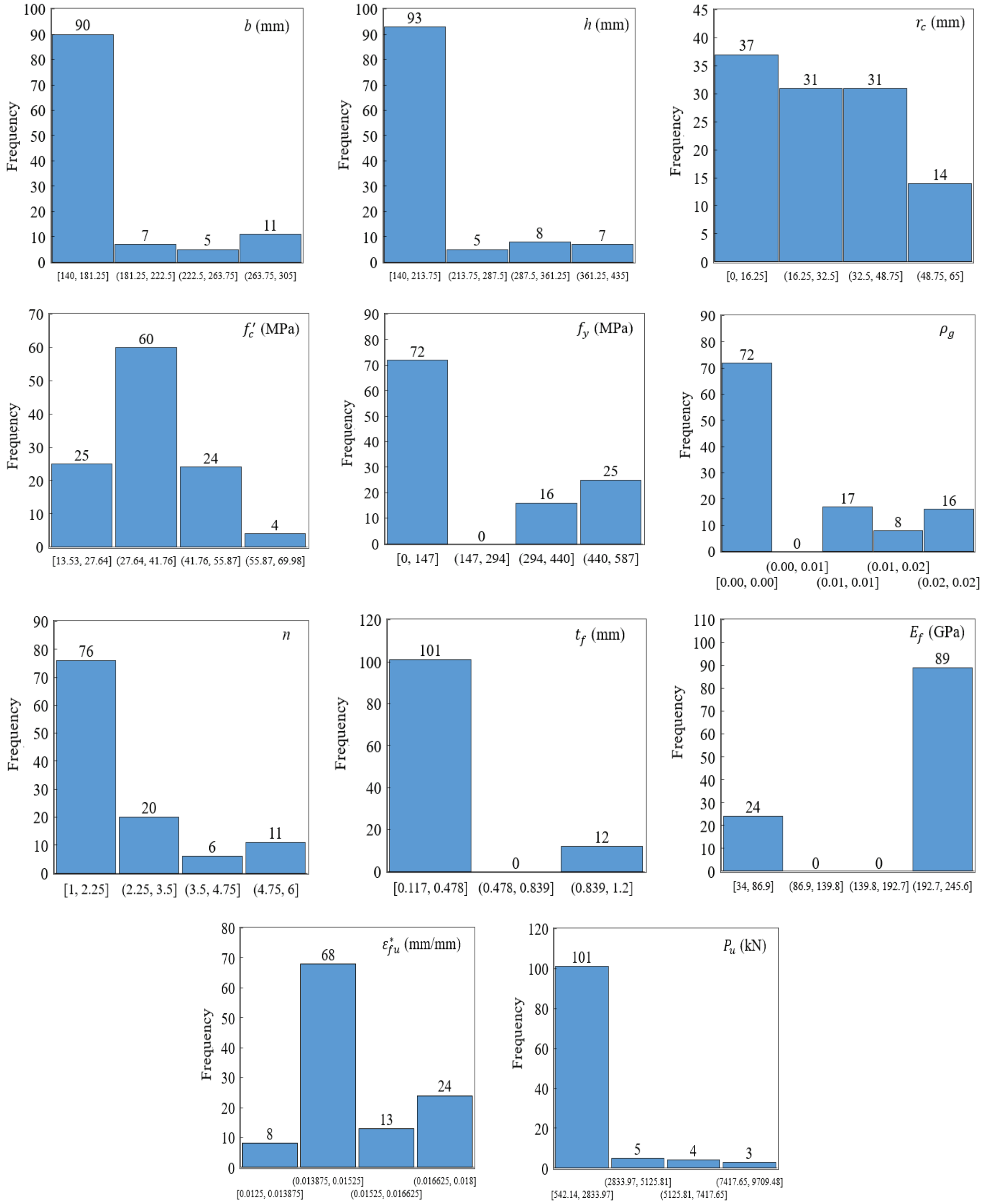


Figure 5. Frequency distributions of the input variables for rectangular test specimens.

the structure of the variables causes a decrease in the axial load carrying capacity of the column [44]. In Figure 6a, it is seen that the mean values of the ϕ for the reinforced column specimens examined in this study are greater than the value recommended in ACI 318-19 (ϕ=0.65) for all cases except the 24rd case. As seen in Figure 6b, the mean values of the ϕ for the unreinforced column specimens are greater than the value recommended in ACI 318-19 (ϕ=0.60) for all cases.

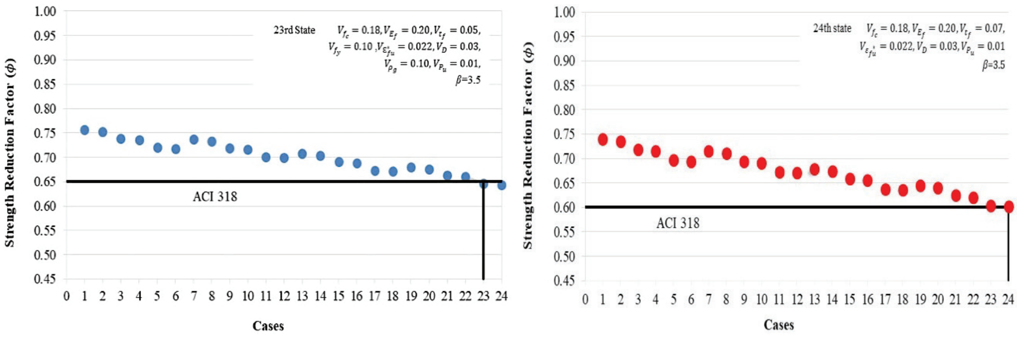
Table 4. Strength reduction factors for reinforced and unreinforced circular test specimens

a) Reinforced

Case	$V_{f'_c}$	V_{E_f}	V_{t_f}	V_{f_y}	$V_{\varepsilon^*_{fu}}$	V_D	V_{ρ_g}	V_{pu}	ϕ
1	0.1	0.1	0.05	0.1	0.022	0.03	0.1	0.01	0.756
2	0.1	0.1	0.07	0.1	0.022	0.03	0.1	0.01	0.752
3	0.1	0.15	0.05	0.1	0.022	0.03	0.1	0.01	0.738
4	0.1	0.15	0.07	0.1	0.022	0.03	0.1	0.01	0.735
5	0.1	0.2	0.05	0.1	0.022	0.03	0.1	0.01	0.720
6	0.1	0.2	0.07	0.1	0.022	0.03	0.1	0.01	0.718
7	0.12	0.1	0.05	0.1	0.022	0.03	0.1	0.01	0.737
8	0.12	0.1	0.07	0.1	0.022	0.03	0.1	0.01	0.733
9	0.12	0.15	0.05	0.1	0.022	0.03	0.1	0.01	0.719
10	0.12	0.15	0.07	0.1	0.022	0.03	0.1	0.01	0.716
11	0.12	0.2	0.05	0.1	0.022	0.03	0.1	0.01	0.701
12	0.12	0.2	0.07	0.1	0.022	0.03	0.1	0.01	0.699
13	0.15	0.1	0.05	0.1	0.022	0.03	0.1	0.01	0.708
14	0.15	0.1	0.07	0.1	0.022	0.03	0.1	0.01	0.704
15	0.15	0.15	0.05	0.1	0.022	0.03	0.1	0.01	0.691
16	0.15	0.15	0.07	0.1	0.022	0.03	0.1	0.01	0.688
17	0.15	0.2	0.05	0.1	0.022	0.03	0.1	0.01	0.673
18	0.15	0.2	0.07	0.1	0.022	0.03	0.1	0.01	0.671
19	0.18	0.1	0.05	0.1	0.022	0.03	0.1	0.01	0.680
20	0.18	0.1	0.07	0.1	0.022	0.03	0.1	0.01	0.676
21	0.18	0.15	0.05	0.1	0.022	0.03	0.1	0.01	0.663
22	0.18	0.15	0.07	0.1	0.022	0.03	0.1	0.01	0.660
23	0.18	0.2	0.05	0.1	0.022	0.03	0.1	0.01	0.646
24	0.18	0.2	0.07	0.1	0.022	0.03	0.1	0.01	0.644

b) Unreinforced

Case	$V_{f'_c}$	V_{E_f}	V_{t_f}	$V_{\varepsilon^*_{fu}}$	V_D	V_{pu}	ϕ
1	0.1	0.1	0.05	0.022	0.03	0.01	0.740
2	0.1	0.1	0.07	0.022	0.03	0.01	0.735
3	0.1	0.15	0.05	0.022	0.03	0.01	0.719
4	0.1	0.15	0.07	0.022	0.03	0.01	0.715
5	0.1	0.2	0.05	0.022	0.03	0.01	0.697
6	0.1	0.2	0.07	0.022	0.03	0.01	0.694
7	0.12	0.1	0.05	0.022	0.03	0.01	0.715
8	0.12	0.1	0.07	0.022	0.03	0.01	0.710
9	0.12	0.15	0.05	0.022	0.03	0.01	0.694
10	0.12	0.15	0.07	0.022	0.03	0.01	0.691
11	0.12	0.2	0.05	0.022	0.03	0.01	0.673
12	0.12	0.2	0.07	0.022	0.03	0.01	0.670
13	0.15	0.1	0.05	0.022	0.03	0.01	0.679
14	0.15	0.1	0.07	0.022	0.03	0.01	0.674
15	0.15	0.15	0.05	0.022	0.03	0.01	0.659
16	0.15	0.15	0.07	0.022	0.03	0.01	0.655
17	0.15	0.2	0.05	0.022	0.03	0.01	0.638
18	0.15	0.2	0.07	0.022	0.03	0.01	0.635
19	0.18	0.1	0.05	0.022	0.03	0.01	0.644
20	0.18	0.1	0.07	0.022	0.03	0.01	0.640
21	0.18	0.15	0.05	0.022	0.03	0.01	0.625
22	0.18	0.15	0.07	0.022	0.03	0.01	0.621
23	0.18	0.2	0.05	0.022	0.03	0.01	0.604
24	0.18	0.2	0.07	0.022	0.03	0.01	0.602



a) Reinforced **b)** Unreinforced

Figure 6. Distributions of strength reduction factor of reinforced and unreinforced circular test specimens [44].

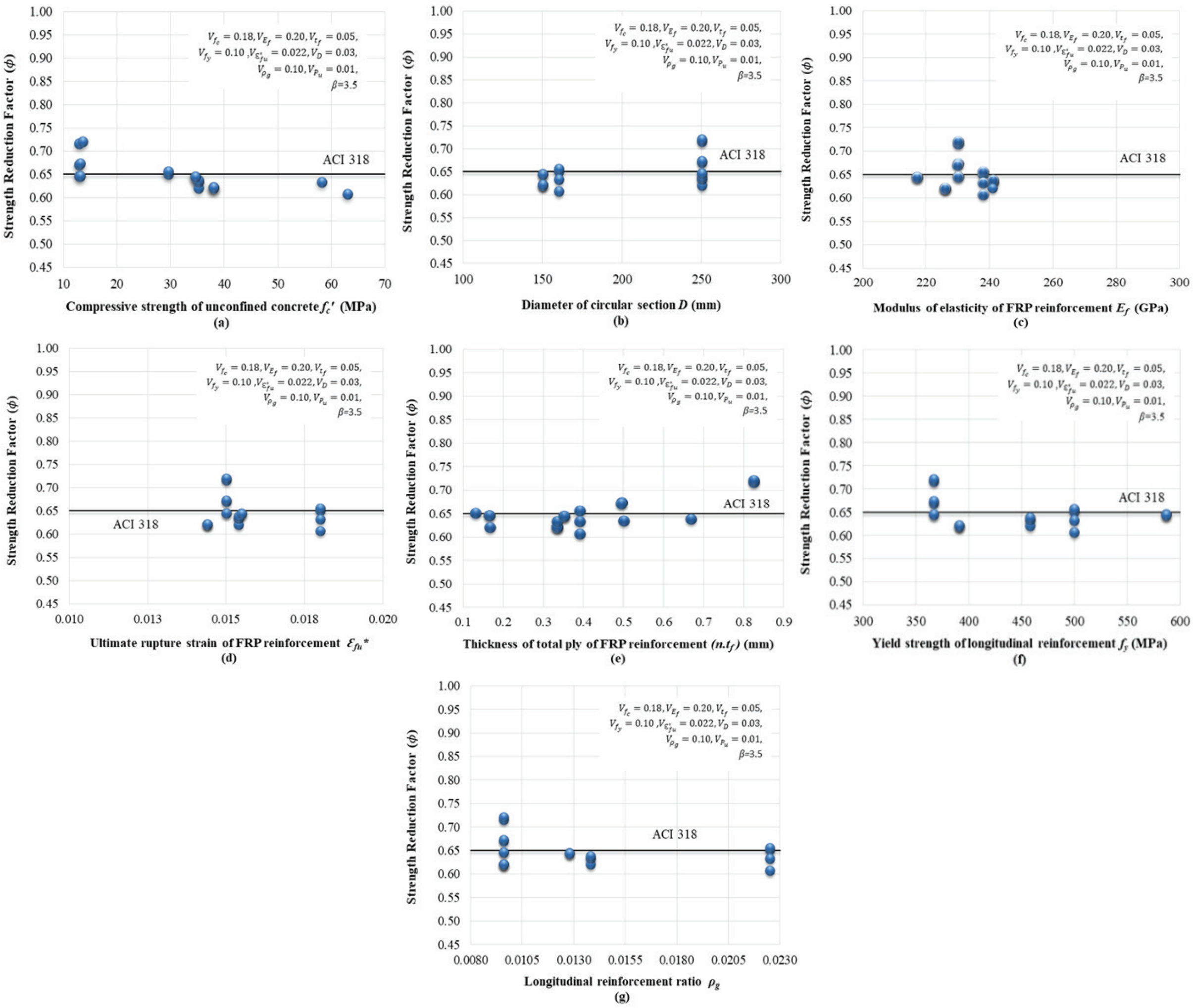


Figure 7. Strength reduction factors versus variables of the reinforced circular columns for case "23" and the target reliability index (β=3.5) [44].

Figures 7 and 8 also present the variations of the strength reduction factors for reinforced and unreinforced columns versus variables constituting the performance function, respectively. When the figures are examined, it could not be concluded that the increment in the values of the variables increases or decreases the strength reduction factor, due to the limited number of column test specimens examined within the scope of the study.

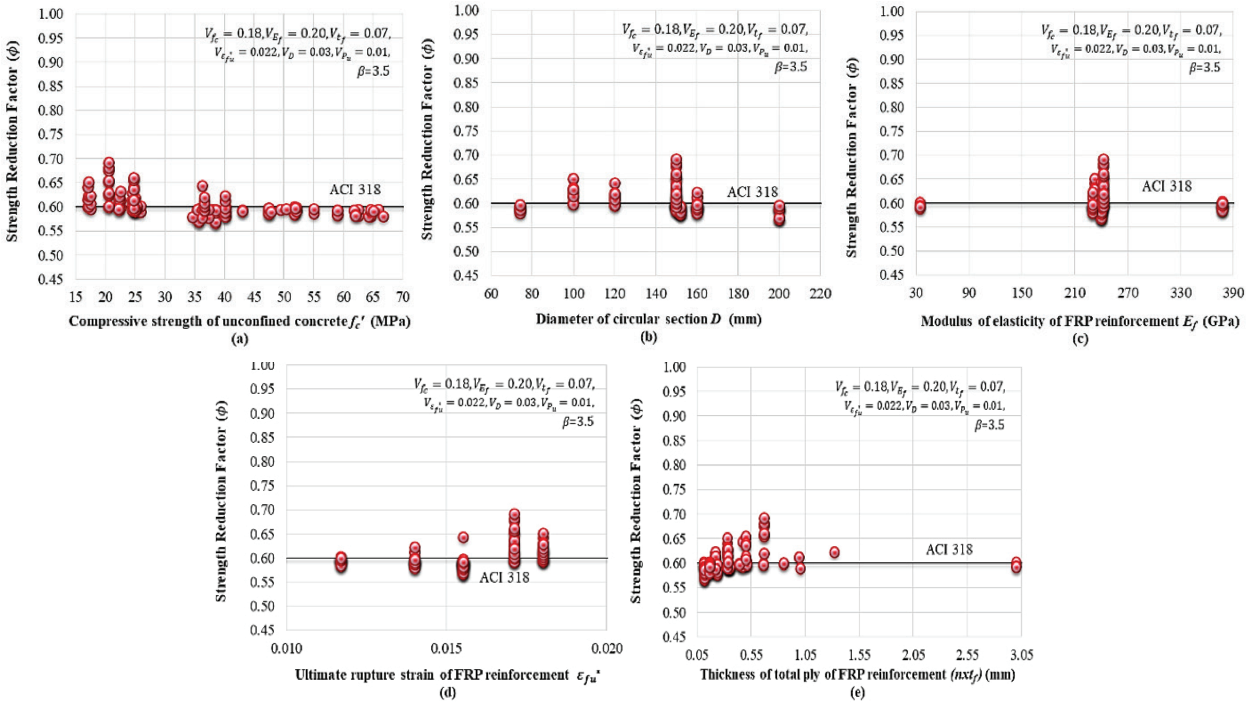
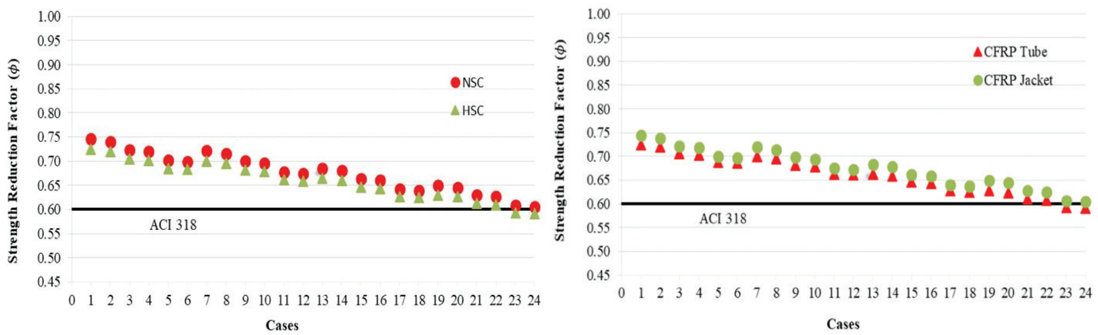


Figure 8. Strength reduction factors versus variables of the unreinforced circular columns for case "24" and the target reliability index (β=3.5) [44].



a) NSC-HSC unreinforced circular columns **b)** CFRP-jacket and CFRP-tube unreinforced circular columns

Figure 9. Comparison of the strength reduction factors of NSC-HSC and CFRP-jacket and CFRP-tube unreinforced circular column specimens for all cases [44].

36 high-strength concrete (HSC) ($f_c' \geq 50$ MPa) and 128 normal-strength concrete (NSC) ($f_c' < 50$ MPa) specimens are considered in this study. The values of the ϕ obtained by taking this distinction into account are compared in Figure 9a. As seen, for all cases, the mean strength reduction factors of the NSC specimens were greater than those of HSC. Also, 141 CFRP-jacket and 23 CFRP-tube confined column specimens are considered. Figure 9b shows that the value ϕ of the specimens confined by CFRP-jacket were greater than those of CFRP-tube columns for all cases.

FRP-confined Rectangular Columns

As seen in Table 5a, (ϕ=0.65) proposed for transversely reinforced rectangular columns in ACI 318-19 corresponds to the case "23" for the (β=3.5). The means of the strength reduction factors ϕ obtained from 41 test data have shown in Table 5a. The strength reduction factors ranged from 0.624~0.739, and in approximately 63% of the specimens, the ϕ is lower than (ϕ=0.65) recommended in ACI 318-19 while approximately 37% of the specimens meet the regulations.

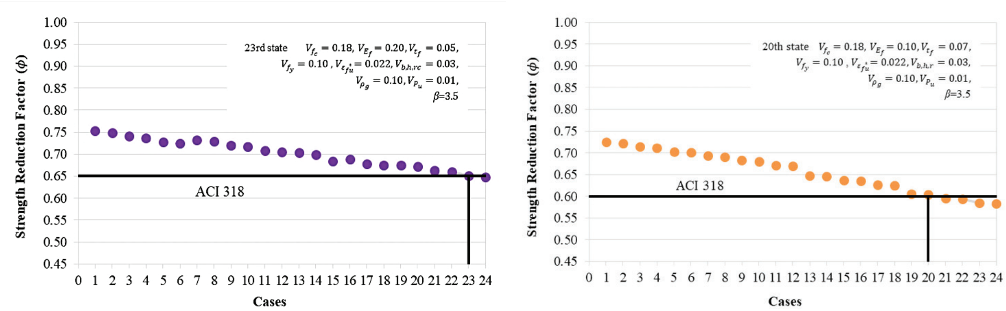
Table 5. Average strength reduction factors for reinforced and unreinforced rectangular test specimens

	Case	$V_{f'_c}$	V_{E_f}	V_{t_f}	V_{f_y}	$V_{\varepsilon^*_{fu}}$	V_b	V_h	V_{r_c}	V_{ρ_g}	V_{pu}	ϕ
a) Reinforced	1	0.1	0.1	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.753
	2	0.1	0.1	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.749
	3	0.1	0.15	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.740
	4	0.1	0.15	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.737
	5	0.1	0.2	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.727
	6	0.1	0.2	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.724
	7	0.12	0.1	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.733
	8	0.12	0.1	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.729
	9	0.12	0.15	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.720
	10	0.12	0.15	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.717
	11	0.12	0.2	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.707
	12	0.12	0.2	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.704
	13	0.15	0.1	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.703
	14	0.15	0.1	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.699
	15	0.15	0.15	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.683
	16	0.15	0.15	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.688
	17	0.15	0.2	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.678
	18	0.15	0.2	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.675
	19	0.18	0.1	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.675
	20	0.18	0.1	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.671
	21	0.18	0.15	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.663
	22	0.18	0.15	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.660
	23	0.18	0.2	0.05	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.650
	24	0.18	0.2	0.07	0.1	0.022	0.03	0.03	0.03	0.1	0.01	0.648

	Case	$V_{f'_c}$	V_{E_f}	V_{t_f}	$V_{\varepsilon^*_{fu}}$	V_b	V_h	V_{r_c}	V_{pu}	ϕ
b) Unreinforced	1	0.1	0.1	0.05	0.022	0.03	0.03	0.03	0.01	0.725
	2	0.1	0.1	0.07	0.022	0.03	0.03	0.03	0.01	0.722
	3	0.1	0.15	0.05	0.022	0.03	0.03	0.03	0.01	0.714
	4	0.1	0.15	0.07	0.022	0.03	0.03	0.03	0.01	0.712
	5	0.1	0.2	0.05	0.022	0.03	0.03	0.03	0.01	0.702
	6	0.1	0.2	0.07	0.022	0.03	0.03	0.03	0.01	0.700
	7	0.12	0.1	0.05	0.022	0.03	0.03	0.03	0.01	0.693
	8	0.12	0.1	0.07	0.022	0.03	0.03	0.03	0.01	0.691
	9	0.12	0.15	0.05	0.022	0.03	0.03	0.03	0.01	0.682
	10	0.12	0.15	0.07	0.022	0.03	0.03	0.03	0.01	0.681
	11	0.12	0.2	0.05	0.022	0.03	0.03	0.03	0.01	0.671
	12	0.12	0.2	0.07	0.022	0.03	0.03	0.03	0.01	0.669
	13	0.15	0.1	0.05	0.022	0.03	0.03	0.03	0.01	0.648
	14	0.15	0.1	0.07	0.022	0.03	0.03	0.03	0.01	0.646
	15	0.15	0.15	0.05	0.022	0.03	0.03	0.03	0.01	0.637
	16	0.15	0.15	0.07	0.022	0.03	0.03	0.03	0.01	0.636
	17	0.15	0.2	0.05	0.022	0.03	0.03	0.03	0.01	0.626
	18	0.15	0.2	0.07	0.022	0.03	0.03	0.03	0.01	0.625
	19	0.18	0.1	0.05	0.022	0.03	0.03	0.03	0.01	0.606
	20	0.18	0.1	0.07	0.022	0.03	0.03	0.03	0.01	0.603
	21	0.18	0.15	0.05	0.022	0.03	0.03	0.03	0.01	0.596
	22	0.18	0.15	0.07	0.022	0.03	0.03	0.03	0.01	0.594
	23	0.18	0.2	0.05	0.022	0.03	0.03	0.03	0.01	0.584
	24	0.18	0.2	0.07	0.022	0.03	0.03	0.03	0.01	0.583

Similarly, the strength reduction factor (ϕ=0.60) proposed for unreinforced columns in ACI 318-19 corresponds to the case “20” for (β=3.5) as seen in Table 5b. The strength reduction factors ϕ for each case were obtained from the 72 test data and ranged from 0.560~0.636, and in approximately 33% of the specimens, the reduction factor is lower than the (ϕ=0.60) recommended in ACI 318-19 while approximately 67% of the specimens meet the regulations [44].

As seen in Figure 10a, the mean values of the ϕ for the reinforced column specimens examined in this study are greater than the proposed value (ϕ=0.65) in ACI 318-19 for all cases except the 24th case. In Figure 10b, it is seen that the mean values of ϕ for the plain column specimens are



a) Reinforced **b)** Unreinforced

Figure 10. Distributions of strength reduction factors for rectangular test specimens [44].

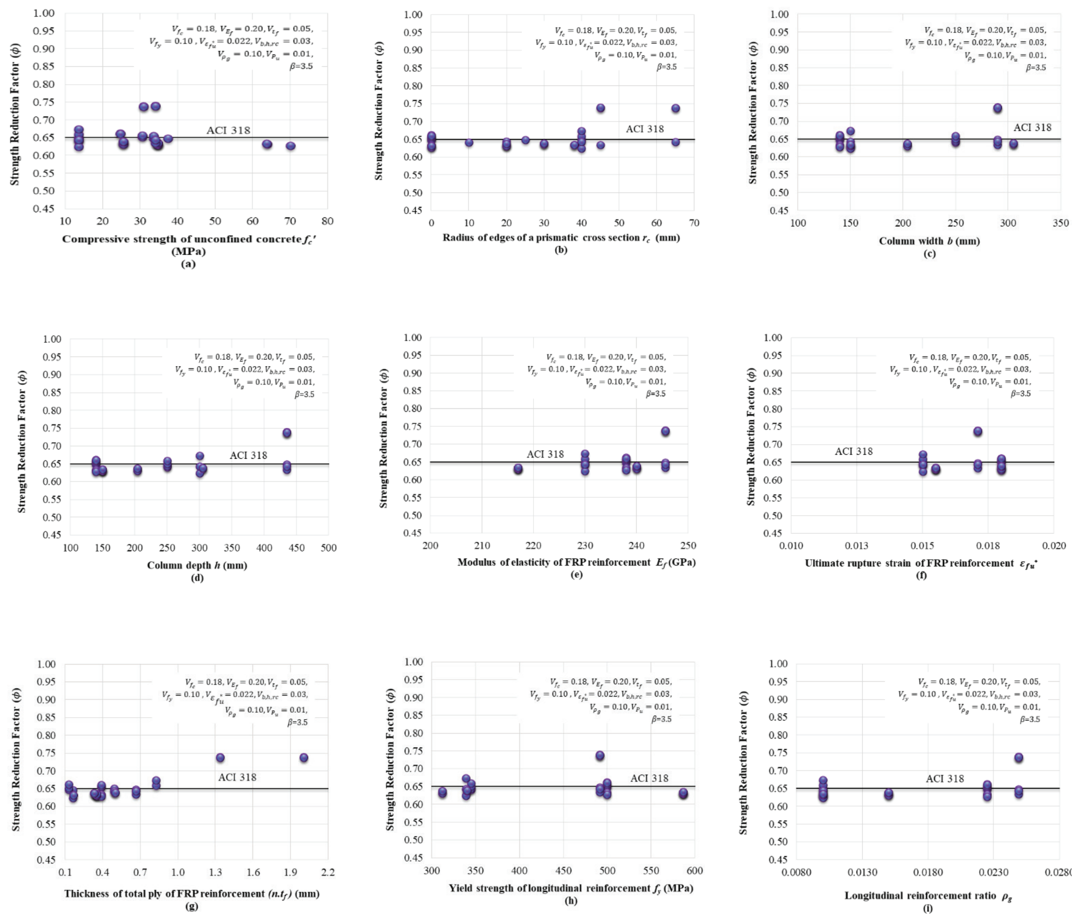


Figure 11. Strength reduction factors versus variables of the reinforced rectangular columns for case “23” and the target reliability index (β=3.5) [44].

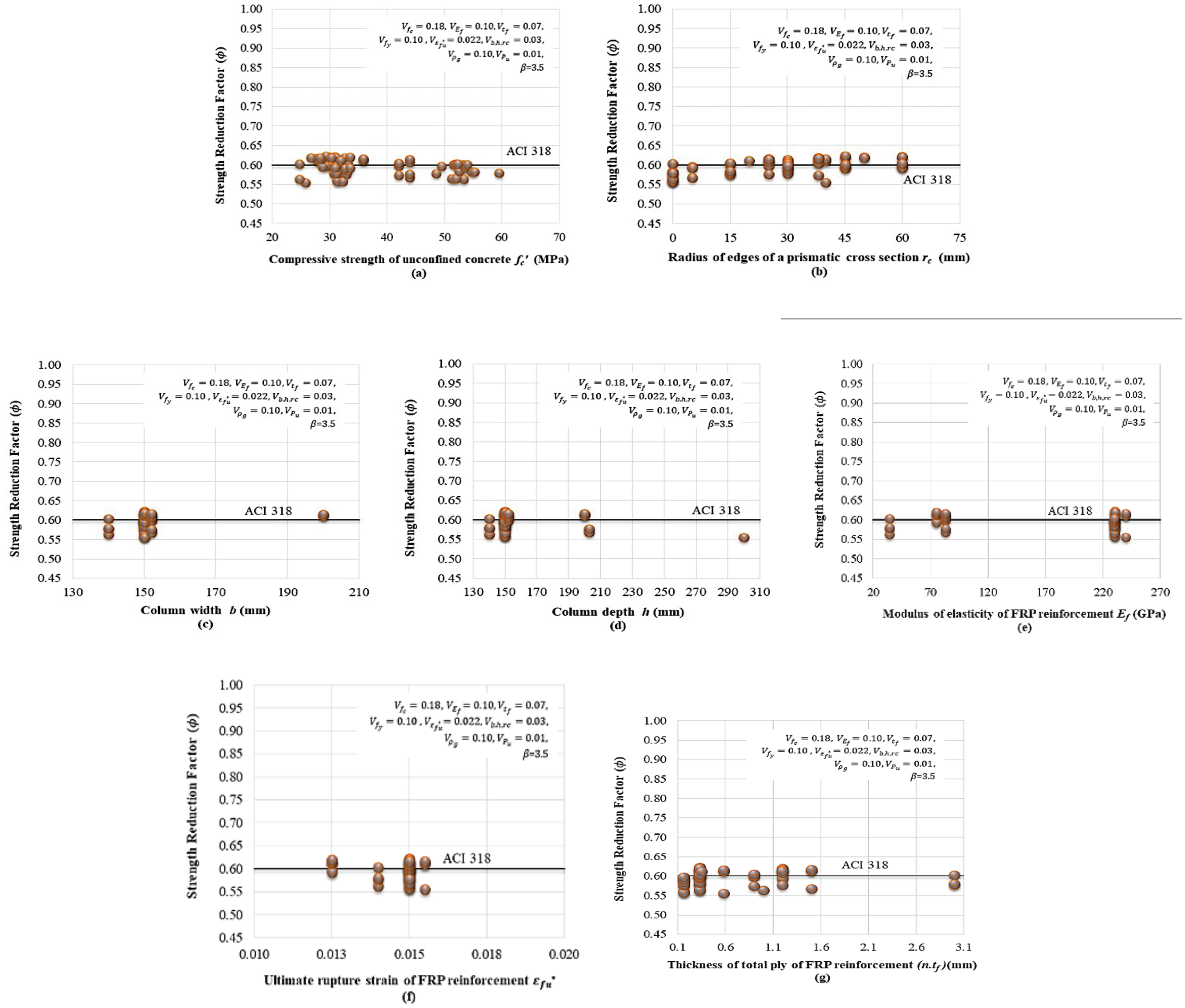


Figure 12. Strength reduction factors versus variables of the unreinforced rectangular columns for case "20" and the target reliability index (β=3.5) [44].

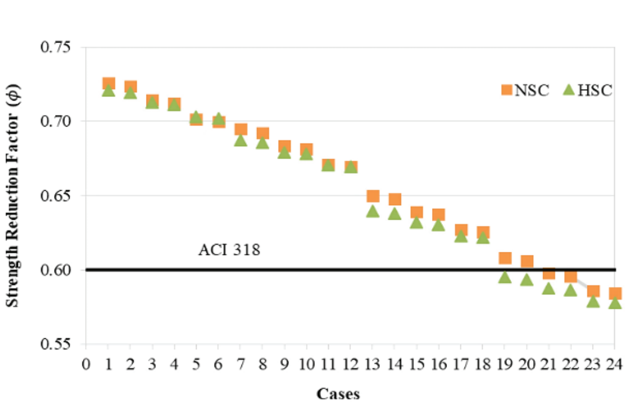


Figure 13. Comparison of the strength reduction factors for NSC and HSC unreinforced rectangular column specimens [44].

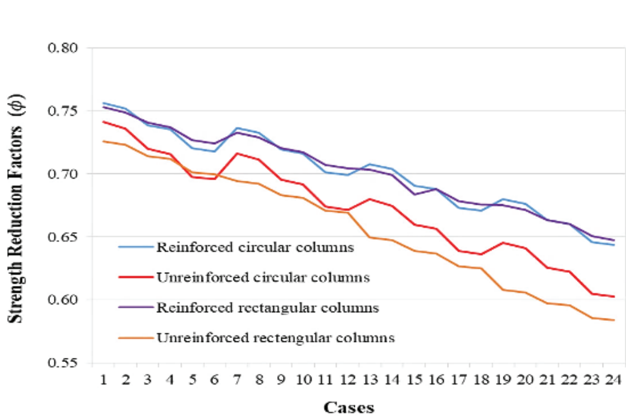


Figure 14. Comparison of the mean strength reduction factors for all column specimens [44].

greater than the proposed value (ϕ=0.60) in ACI 318-19 for all cases except the 21th, 22th, 23rd and 24th case.

As Figures 11 and 12 are examined, it could not be concluded that the increase in the values of the variables in rectangular columns, similar to the circular columns, increases or decreases the strength reduction factor.

Figure 13 shows a comparison of strength reduction factors for NSC and HSC unreinforced rectangular column specimens. As seen in Figure 13, the mean strength reduction factors ϕ of the NSC specimens were greater than those of HSC specimens except in cases 5 and 6. The differences between NSC and HSC columns for cases 5 and 6 are about 0.002.

In Figure 14, the mean ϕ values for the different columns examined in this study are compared. From the examination of the Figure 14, it is seen that ϕ values for FRP-confined unreinforced columns are lower than those of FRP-confined reinforced columns for all cases. In circular and rectangular cross-section columns with longitudinal reinforcement, the ϕ is obtained higher than the unreinforced columns, since the effect of longitudinal reinforcement increases the axial capacity of the column. The figure shows the comparison of the calculated mean ϕ values for the different columns examined in the study.

CONCLUSION

This study evaluates the strength reduction factors ϕ to provide the target reliability index (β=3.5) of the CFRP-confined circular and rectangular columns under axial compression load. These factors were determined by probabilistic methods for 298 column specimens collected from the literature, according to 24 different coefficients of variation of the variables. Then, obtained factors were compared with ACI 318-19 ones. To provide (β=3.5), the extreme values of coefficients of variation were investigated. The following conclusions can be obtained from the current study:

- In all columns, the strength reduction factors are inversely proportional to the coefficients of variation of the variables constitute the performance function. The strength reduction factors are decrease with the increase of the coefficients of variation of the variables. Namely, an increase in the coefficient of variation causes a decrease in the reliability of the column.
- Since the effect of reinforcement increases the axial load capacity, the strength reduction factors (ϕ) in the reinforced columns are higher than the coefficients in the unreinforced columns.
- In all column specimens, it is observed that the effect on ϕ of the uncertainty in the compressive strength of the concrete ($V_{f_c'}$) is the highest compared to other variables.
- The strength reduction factors (ϕ) of the (NSC) specimens were obtained higher than those of (HSC) specimens for circular columns. Similarly, the strength reduction factors (ϕ) of the (NSC) specimens for rectangular columns were found to be higher except for cases 5 and 6. It can be interpreted that since the (HSC) specimens are more brittle materials compared with (NSC), the reduction factors of the (HSC) specimens were obtained smaller than (NSC) ones. However, further research should be conducted to verify the results due to the limited number of test specimens for (HSC) columns.
- The sensitivity of the results obtained in a probabilistic study is closely related to the adequacy of the database considered in the research. The above-obtained results are only valid for the specimens collected from the literature. For this reason, it is necessary to examine more specimens with different materials and qualities to determine the strength reduction factors more sensitively.

NOMENCLATURE

A_c	Cross-sectional area of column (mm^2)
A_e	Effective confinement area A_e (mm^2)
A_g	Gross area of concrete section (mm^2)
A_{st}	Total area of longitudinal reinforcement (mm^2)
b	Cross-sectional short side dimension of column (mm)
C_E	Environmental reduction factor
D	Diameter of circular section or diagonal distance equal to $D = \sqrt{b^2 + d^2}$ of the rectangular section (mm)
E_f	Modulus of elasticity of FRP reinforcement (MPa)
f_c'	Compressive strength of unconfined concrete (MPa)
f_{cc}'	Compressive strength of FRP-confined concrete (MPa)
f_l	Maximum confinement pressure of FRP jacket (MPa)
f_y	Yield strength of longitudinal reinforcement (MPa)
$g(x)$	Performance function
h	Cross-sectional long side dimension of column (mm)
L	Height of compression member (mm)
m_i	Mean value of random variable
m_{xi}^N	Mean value for equivalent normal distribution of x_i
n	Number of plies of FRP reinforcement
p_F	Failure probability
P_n	Nominal axial compressive strength (kN)
P_u	Ultimate axial compressive strength (kN)
r_c	Radius of edges of a prismatic cross section confined with FRP (mm)
t_f	Nominal thickness of one ply of FRP reinforcement (mm)
V_i	Coefficient of variation of random variable
x_i	Random variable
x_i^*	Most probable failure point

Greek symbols

α Sensitivity coefficient

β Reliability index

ε_{ccu} Ultimate axial strain of confined concrete

ε_{fe} Effective strain in FRP reinforcement attained at failure

ε^*_{fu} Ultimate rupture strain of FRP reinforcement

ε_{fu} Design rupture strain of FRP reinforcement

ϕ Strength reduction factor

γ Safety factor

κ_a Shape factor

κ_b Efficiency factor for FRP reinforcement in determination of ε_{ccu}

κ_ε Strain efficiency factor

ρ_g Longitudinal reinforcement ratio

σ_i Standard deviation of random variable

σ^N_{xi} Standard deviation for equivalent normal distribution of x_i

Ψ_f Additional reduction factor

AUTHORSHIP CONTRIBUTIONS

Authors equally contributed to this work.

DATA AVAILABILITY STATEMENT

The authors confirm that the data that supports the findings of this study are available within the article. Raw data that support the finding of this study are available from the corresponding author, upon reasonable request

CONFLICT OF INTEREST

The author declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

ETHICS

There are no ethical issues with the publication of this manuscript

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